Change of ground type by means of dynamic compaction: consequence on the calculation of seismic loadings

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Abstract Soil improvement works by means of dynamic compaction or vibrocompaction improves mechanical characteristics of granular soils. Experience feedbacks point out the geotechnical conditions which are able to induce ground type change after ground improvement works. In this paper, in case of a two-layer geotechnical model and dynamic compaction technique, the necessary conditions are presented for a ground type change and the consequences on the magnitude of the seismic design loadings are studied for a five-storey building. Therefore, this article does not deal with liquefaction but only with seismic retro-analysis after soil densification.

Keywords Dynamic compaction · Seismic loadings · Ground types

Introduction

This article presents the geotechnical conditions convenient to the change of the initial ground type, preliminary defined according to the European Standard EN 1998 [7] for design structures for earthquake resistance, after the works of densification of granular soil by methods such as

dynamic compaction or vibrocompaction. To date, the ground densification leading to such performance is clearly not the objective of the treatment, but could become it, in particular if it brings an economic advantage in term of reinforced concrete for building projects. In fact, changing the type of the soil reduces the global seismic loads calculated from horizontal design response spectra. Liquefiable soils and the liquefaction mitigation by densification techniques are not studied here.

International ground types

The analysis of the soils' mechanical characteristics before and after treatment could indeed lead to hold a different ground elastic spectrum [3, 5], according to international standards for soil classification in case of seismic hazard (UBC, Eurocode 8, etc.). Figure 1 presents a synthesis of soil types defined in European Standards [7], Uniform Building Code [11] and Building Center of Japan [1].

It is well known that ground densification could severely increase the mechanical characteristics of the soils tested with cone penetration test—CPT or pressure meter test—PMT [15]. To rule on the relevance of an effective change of class of ground, the classical site investigation is not sufficient. In fact, it is necessary to prefer shear wave velocity measurement (V_s) more adapted to the soil small strain parameters inferred by seismic waves ($\varepsilon \ll 10^{-4}$). Most of the worksites in Western Europe are concerned with a small strain seismicity [21]. In Fig. 1, one can note that even if the name of each type of soil is different, the $V_{s,30}$ limits are close or similar [20]. This article is limited to the ground types of Eurocode 8. Soil classes are defined with the harmonic mean, formula (1), calculated for 30 m of soil below the ground surface.

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UBC	$V_{s,30}$ (m/s)				
	200	350	750	1 500	
	E	D	C	B	A
Eurocode 8	$V_{s,30}$ (m/s)				
	100	180	360	800	
		D	C	B	A
BCJ	$V_{s,30}$ (m/s)				
	$(T > 0.6 \text{ s}) 200^*$		$(T < 0.2 \text{ s}) 600^*$		
	III		II		I

Fig. 1 Comparison of soil classification based on $V_{s,30}$ (Uniform Building Code [11]; Eurocode 8 [7], Building Center of Japan [1])

Fig. 2 Ground densification by means of dynamic compaction (a), vibrocompaction (b) [5]. Eurocode 8: case of ground type change (D to C) after soil improvement (c) [16]

$$V_{s,30} = \frac{30}{\sum_{i=1}^n \frac{h_i}{V_{si}}} \quad (1)$$

Field of application

The ground improvement techniques described here are dynamic compaction and vibrocompaction (Fig. 2) essentially for granular soils. In both cases the ground densification is analysed from 0 to 20 meters thick and the reason is exposed in the next section because an initial E soil type is studied. It could be a treatment to mitigate either an insufficient soil bearing capacity either incompatible settlements for the project. Vibrocompaction is a method using a vibratory probe inserted into the ground. It was carried out to a depth of more than 60 m for onshore and offshore projects. The method is mainly applied to the densification of hydraulic sandfill with various carbonate contents [6]. Dynamic compaction consists on dropping a

heavy weight from air onto ground. Dynamic compaction and vibrocompaction are classified as “ground improvement without admixtures in non-cohesive soils or fill materials” according to the TC 17 ground improvement meeting (www.bbri.be/go/tc17).

The soils in this study are mostly sands and gravels. The efficiency of vibrocompaction techniques is optimum if the fine content (passing through the n°200 sieve or grain size smaller than 75 μm) does not exceed 10–15 % of the total weight of the soil sample [19] and less than 2 % of clay (grain size smaller than 2 μm). Some authors suggest using CPT results to judge the suitability of the vibrocompaction method [17].

Geotechnical model and V_s values

To realize our demonstration of ground type change, let consider the model described in Fig. 3. This model is compounded of two layers of earth materials: soil 1 and

Fig. 3 Eurocode 8 ground types (a). Two layers' model (b). Soil 1 is D or C type, with V_{s1} shear wave velocity and thickness $h_1 > 5$ m. Soil 2 is the seismic bedrock, with velocity V_{s2} greater than 800 m/s and thickness $H-h_1$. The total thickness is $H = 30$ m for the calculation of $V_{s,30}$. x is the thickness of densified soil

Ground type and description of stratigraphic profile – Eurocode 8
A: Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface. $V_{s,30} > 800$ m/s.
D: Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. $V_{s,30} < 180$ m/s.
E: A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.
S_1, S_2 : Soils potentially concerned with liquefaction (see EN 1998).

soil 2. Soil 1 is defined as alluvial and/or glacial detrital deposits. The shear wave velocity of soil 1, named as V_{s1} , is lower than 360 m/s (i.e., C or D ground type according to Eurocode 8) and its thickness is h_1 . Soil 2 is assimilated to geotechnical and seismic bedrock (A ground type) and so, V_{s2} is upper than 800 m/s. The objective of the following lines is to evaluate $V_{s,30}$, the harmonic mean of shear wave velocity for the upper $H = 30$ m of soil, and to define the ranges of variation $\{V_{s1}, V_{s2}\}$ and h_1 that permit to change from a soil type (E) to another (A). The right definition of E soil is a surface alluvium layer with V_s values of type C or D and thickness varying from 5 m to 20 m, underlain by stiffer material with $V_s > 800$ m/s.

Ground type change by means of the dynamic compaction technique

Thanks to various studies led on work sites [2, 5, 10–13] and different geophysical investigation techniques (multi-channel analysis of surface waves, horizontal-to-vertical spectral ratio, etc.), the assumption is made that a typical ground densification program could lead to improve the shear wave velocity of soil 1, V_{s1} , of more than 30 %. The study focuses on dynamic compaction works [2, 5] and the case study of 30 % of increase is explored. It should be noted that the increase of V_{s1} is mainly not in the scope of work. To date, it is only secondary consequence, especially if the problem of bearing capacity or/and settlement mitigation had been solved elsewhere. For ground densification specialists, it is known that it is not necessary to obtain severe settlement to reach the objective of improvement of the Young's modulus [18]. In fact, a volume reduction of 3–5 % of granular soil could double the modulus value. By

Fig. 4 Relative horizontal design response spectra $S_d(T)$ normalized by a_g (5 % damping) for a E and A ground type (spectra type 2) with ductile behaviour $q = 1.5$. The term a_g is the design ground acceleration on type A ground. The period of the building (T_{building}) is plotted in the figure as well as the range of possible values for the fundamental period of the soil (T_{soil})

this remark, the capacities of the technique are not totally explored for a significant increase of shear wave velocity. The main reason is the ground vibrations generated by the dynamic compaction technique which is however more and more used in urbanized areas [4]. The Fig. 4 shows the difference between design response spectra calculated for A and E ground types according to Eurocode 8 standards.

Assuming that some soil improvement works make the soil shift from E to A type, the design of the structure could be adjusted and/or optimized. In this case study, one can note that the range of variation of the fundamental period of the ground ($0.055 < T_{\text{soil}} < 0.44$ s) do not overlap the period of the building for the first calculated mode

Fig. 5 Permitted values (grey zone) for the change of ground type (E to A). The shear waves velocity of the seismic substratum is $V_{s2} = 1500$ m/s and the shear waves velocity of soil 1, is in the range $V_{s1 \text{ initial}} \in [180, 260]$ m/s for $h_1 = 5$ to 12 m and 30 % of shear waves velocity increase after dynamic compaction, i.e., $V_{s1 \text{ final}} \in [240, 360]$ m/s]

Fig. 6 Domain of validity for the change of ground type (E to A). The shear waves velocity of the seismic substratum is $V_{s2} = 1200$ m/s. The only possible case of ground type change is limited to the arrow in the graph ($h_1 = 5$ m), along the y-axis, in case of 30 % of shear waves velocity increase after dynamic compaction. The initial $V_{s,30}$ is around 700 m/s

Fig. 7 Domain of validity for the change of ground type (E to A). x is the thickness of the treatment

($T_{\text{building}} = 0.63$ s), i.e., the case of resonance is prevented, even after ground densification works. Here, the assumptions considered are $V_{s1} \in [180, 360]$ m/s and $h_1 \in [5, 20]$ m/s]. In Figs. 5, 6 and 7, the permitted values $\{h_1, V_{s1}\}$ are presented for a soil type change E to A. Figure 5 shows, for $V_{s2} = 1500$ m/s, that it is possible to move from E to A type if the initial $V_{s,30}$ is greater than 700 m/s. The critical case is obtained for $V_{s2} = 1200$ m/s, i.e., the only possible case of ground type change is limited to $h_1 = 5$ m. If $V_{s2} < 1200$ m/s there is no possibility of modification except a higher proportion of V_{s1} improvement ($>30\%$).

Another way of presenting the results consists in drawing the domains of validity, according to x (Eq. 2). x is the thickness of compacted soil (Fig. 7). For $h_1 = 5$ m and $V_{s2} = 1500$ m/s, it works when $V_{s1 \text{ initial}} \in [185, 240]$ m/s with 30 % of shear wave velocity increase after dynamic compaction. For $h_1 = 5$ m and $V_{s2} = 1000$ m/s, it works when $V_{s1 \text{ initial}} \in [310, 360]$ m/s and for $h_1 = 10$ m and $V_{s2} = 1500$ m/s, it works when $V_{s1 \text{ initial}} \in [320, 360]$ m/s, always with 30 % of shear wave velocity increase after dynamic compaction. For $h_1 = 10$ m and $V_{s2} > 1250$ m/s and for $h_1 > 11$ m and any V_{s2} , there is no possibility of modifying except a higher proportion of V_{s1} improvement ($>30\%$).

The condition to observe a type change (E to A) is described by the inequality below (2):

$$x \geq \frac{a}{1-a} \left(\frac{30 \times V_{s1}}{V_{\min,A}} - h_1 - \frac{V_{s1} \times (30 - h_1)}{V_{s2}} \right) \quad (2)$$

with $a = 1.30$ the improvement factor (here limited to 30 % for V_{s1}) and $V_{\min,A} = 800$ m/s the minimum V_s for a A soil type.

Calculation of seismic loadings: seismic analysis of a five-storey building by modal analysis

A regular five-storey building is studied in this paragraph. The geometrical characteristics of this structure are described below (Fig. 8):

- length of the supporting walls: $l_w = 6$ m; thickness of the supporting walls: $b_w = 0.3$ m;
- modulus of elasticity of the concrete: $E_{cm} = 26$ GPa;
- constant floor height: $h = 4$ m;
- constant mass for each floor and for one supporting wall: $m = 300$ t;
- behavior coefficient (Eurocode 8): $q = 1.5$; the behavior coefficient is introduced in seismic based design as a corrective coefficient to reduce the acceleration defined from a spectrum built on the base of elasticity, to take the positive effect of ductility.

Fig. 8 Case study of a five-storey building (a) modelled by means of five oscillators of one degree of freedom (b). Here, the seismic disturbance is $x_g(t)$ applied at the basement of the building (adapted from [14])

The building is modelled as an 1D “skewer” by means of a set of five non-heavy beams and of five masses [14]. The matrix equation of motion take the form of a dynamic three-dimensional spring mass system (3):

$$\underline{M} \times \underline{\ddot{x}} + \underline{C} \times \underline{\dot{x}} + \underline{K} \times \underline{x} = -\underline{M} \times \underline{e_x} \times x_g(t) \quad (3)$$

where $\underline{M}$ is the mass matrix, $\underline{C}$ is the damping matrix, $\underline{K}$ is the stiffness matrix and x_g is the soil acceleration. The displacement, velocity and acceleration vectors are, respectively, $\underline{x}$, $\underline{\dot{x}}$ and $\underline{\ddot{x}}$.

The equations system is coupled. It can be decoupled using modal coordinates. The eigenmodes ω_n which are the eigenvalues of the system are the natural modes of the structure. They are obtained by nullifying the determinant of the system (4):

$$|\underline{K} - \omega_n^2 \times \underline{M}| = 0 \quad (4)$$

The building periods are linked to the eigenmodes by formula (5):

$$T = \frac{2\pi}{\omega_n} \quad (5)$$

The relative displacements (x_j) are transformed to modal coordinates or generalized coordinates (z_n) by changing variables (6):

$$\underline{x} = \underline{A} \times \underline{z} \quad (6)$$

with $\underline{A}$ the modal vectors (or eigenvectors) matrix and $\underline{z}$ the modal coordinates vector.

After changing variables, the equations system is transformed to modal equations system. For the n th line (7):

$$z_n + 2 \times \zeta_n \times \omega_n \times \dot{z}_n + \omega_n^2 \times z_n = -\frac{r_n}{m_n^*} \times x_g(t) \quad (7)$$

where $\frac{r_n}{m_n^*}$ is the modal participation factor and ζ_n is the damping coefficient of the structure.

The participation factor vector is defined by (8):

$$\underline{r} = \underline{A}^T \times \underline{M} \times \underline{e_x} \quad (8)$$

The effective modal mass $m_{\text{mod},n}$ for mode n quantifies the contribution of this mode to the total response (9):

$$m_{\text{mod},n} = \left(\frac{r_n}{m_n^*} \right)^2 \times m_n^* = \frac{\left(\sum_{j=1}^N A_{jn} \times m_j \right)^2}{\sum_{j=1}^N A_{jn}^2 \times m_j} \quad (9)$$

Note that the first mode has a much higher effective modal mass than the second mode, etc.

From the ground design response spectrum, the maximum modal response can be determined by the Eq. (10):

$$z_{n,\text{max}} = \frac{|r_n|}{\omega_n^2 \times m_n^*} \times S_{d,n}(\omega_n, \zeta) \quad (10)$$

And then for each mode, the maximum forces are determined by (11):

$$\begin{aligned} F_{n,\text{max}} &= \underline{K} \times \underline{x}_{n,\text{max}} = \underline{K} \times \underline{A}_n \times z_{n,\text{max}} \\ &= \underline{K} \times \underline{A}_n \times \frac{|r_n|}{\omega_n^2 \times m_n^*} \times S_{d,n}(\omega_n, \zeta) \end{aligned} \quad (11)$$

Fig. 9 Impact of the change of ground type on the normalized maximum design bending moment for a regular five-storey building by considering all retained hypotheses of the model (height, mass, supporting walls, behavior coefficient). These curves are the same for the normalized maximum shear force, plus or the minus five percent, in this particular case. They are independent of the building category

Eventually, the modal analysis needs to calculate the magnitude of forces by means of combination methods. Here, the square-root-of-the-sum-of-the-squares (SRSS) method is used to determine the maximum relative displacements ($x_{\max,tot}$), the maximum forces ($F_{\max,tot}$), the maximum shear forces ($V_{\max,tot}$) and the maximum bending moments ($M_{\max,tot}$). Here, the maximum relative displacement $x_{\max,tot}$ is calculated with Eq. (12):

$$x_{\max,tot} = \sqrt{\sum x_{n,\max}^2} \quad (12)$$

Results of the modal analysis of our example and application to various ground types

The results in terms of normalized maximum bending moments are presented (Fig. 9) for types 1 and 2 spectra. The maximum value is obtained, in this case, for the D soil. It is to note that these curves are the same for the normalized maximum shear force, plus or the minus five percent. They are independent of the building category, when expressed in normalized and relative values.

According to the Eurocode 8, for type 1 spectra, if the ground type changes from E to A, the maximum bending moment and the maximum shear force decrease by 43 % and by 14 % if the ground changes from E to B. If the ground type changes from D to C, they decrease by 19 %. For the type 2 spectra, if the ground type changes from E to A, the maximum bending moment and the maximum shear force decrease by 37 % and by 16 % if the ground changes from E to B. If the ground type changes from D to C, they decrease by 30 %.

Discussion on perspectives

The required soil conditions are illustrated to allow a change of the ground type, with a standards point of view (Eurocode 8). Most of new generation seismic codes propose, for the flat-rate method, threshold values between each ground type, with the objective to define the acceleration spectrum used for the design of a building, i.e., the calculation of the seismic loads. The flat-rate approach could lead to the singular situation of a change of the ground type. Recent experiments on worksite had shown that the increase of the shear wave velocity V_{s1} , and the mean value $V_{s,30}$ for 30 m of upper soils, could represent tens of percent. In this paper, the percentage of 30 % is kept. The change from D to C, C to B and, in more restrictive conditions, E to A, seems realistic, subject however to specific geological geometry and mechanical properties.

The impact of a ground type change on the design of a building after densification of granular soils by means of dynamic compaction techniques could be significant and benefic in terms of load amplitude reduction. It could be an economic asset but other aspects have to be controlled to definitely validate this retrofitting approach for the seismic design. In fact, the ground densification could modify the fundamental period ($T = 4H/V_s$) of the soil model by acting on V_s thus the case of resonance must be systematically controlled. This aspect is not observed here due to the underlying assumptions. Another antagonist feature could be the modification of the damping ratio of the initial soil. It is difficult to estimate this parameter with in situ techniques but its impact is significant on calculations.

Another important perspective of these results is the study of the impact of the soil improvement on dynamic impedance functions ($K = f(\rho, V_s, \nu)$) used for surface foundations. In fact, one of the fundamental problems in dynamic soil-structure interaction is the characterization of the dynamic response of surface foundations resting on a soil medium under time-dependent loads. For each particular harmonic excitation with pulsation ω , the dynamic impedance function is defined as the ratio between the steady-state force and the resulting displacement at the base of the massless foundation [8, 9].

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